MASTER'S THESIS ON

APPLICATION OF NON-LINEAR PUSHOVER ANALYSIS FOR SEISMIC PERFORMANCE EVALUATION OF EXISTING HIGH RISE BUILDINGS IN ETHIOPIA BY USING SAP2000 (CASE STUDY)

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ABSTRACT

[**Keywords**: Non-linear static procedure; reinforced concrete frame; pushover analysis; target displacement; yield strength; pushover curve].

With the immense loss of life and property witnessed in the last couple of decades alone in various parts of the world, due to failure of structures caused by earthquakes, attention is now being given to the evaluation of the adequacy of strength in framed RC structures to resist strong ground motions. A 15-year old 4 story (8-bay and 3-frame) reinforced concrete structure has been considered in this study, which lies in Walita Sodo. Masonry infill's have been considered as non-structural members during this entire study. Inelastic static analysis, or pushover analysis, has been the preferred method for seismic performance evaluation due to its simplicity. It is a static analysis that directly incorporates nonlinear material characteristics. Inelastic static analysis procedures include Capacity Spectrum Method, Displacement Coefficient Method and the Secant Method. The structure has been evaluated using Pushover Analysis, a non-linear static procedure, which may be considered as a series of static analysis carried out to develop a pushover curve for the building. The structure analyzed by using SAP-2000, after being designed in STAAD.Pro v8i by considering M20 concrete and Fe 250 steel reinforcement. The pushover curve is generated by pushing the top node of structure to the limiting displacement and setting appropriate performance criteria. The target displacement for the structure is derived by bi- linearization of the obtained pushover curve and subsequent use of Displacement Coefficient Method according to ASCE 41-06.The analysis is then carried out for 150% of the calculated target displacement for the structure to observe the yielding of the members and the adequacy of the structural strength. The extent of damage experienced by the structure at the target displacement is considered representation of the damage that would be experienced by the building when subjected to design level ground shaking.

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Chapter 1

INTRODUCTION

1. INTRODUCTION

1.1. GENERAL

The term **earthquake** can be used to describe any kind of seismic event which may be either natural or initiated by humans, which generates seismic waves. Earthquakes are caused commonly by rupture of geological faults; but they can also be triggered by other events like volcanic activity, mine blasts, landslides and nuclear tests. An abrupt release of energy in the Earth's crust which creates seismic waves results in what is called an earthquake, which is also known as a tremor, a quake or a temblor). The frequency, type and magnitude of earthquakes experienced over a period of time defines the seismicity (seismic activity) of that area. The observations from a seismometer are used to measure earthquake. Earthquakes greater than approximately 5 are mostly reported on the scale of moment magnitude. Those smaller than magnitude 5, which are more in number, as reported by the national seismological observatories are mostly measured on the local magnitude scale, which is also known as the Richter scale.[1]

There are many buildings that have primary structural system, which do not meet the current seismic requirements and suffer extensive damage during the earthquake. The buildings at Walita sodo were designed by primary structural system and the reason behind this is Wolita Sodo lies in ZONE IV of Seismic Zone Map, which says the region is most probable for earth quakes. Recently, buildings around there are committing lateral effect damages. The building is a Four story building designed. At present time the methods for seismic evaluation of seismically deficient or earthquake damaged structures are not yet fully developed. [1]

The buildings which do not fulfill the requirements of seismic design, may suffer extensive damage or collapse if shaken by a severe ground motion. The seismic evaluation reflects the seismic capacity of earthquake vulnerable buildings for the future use. [1]

According to the Seismic Zoning Map of Ethiopia, It is divided into four zones on the basis of seismic activities. Walita sodo lies in Zone IV as shown in the map below based on the Ethiopian Building Codes (EBCS 8).

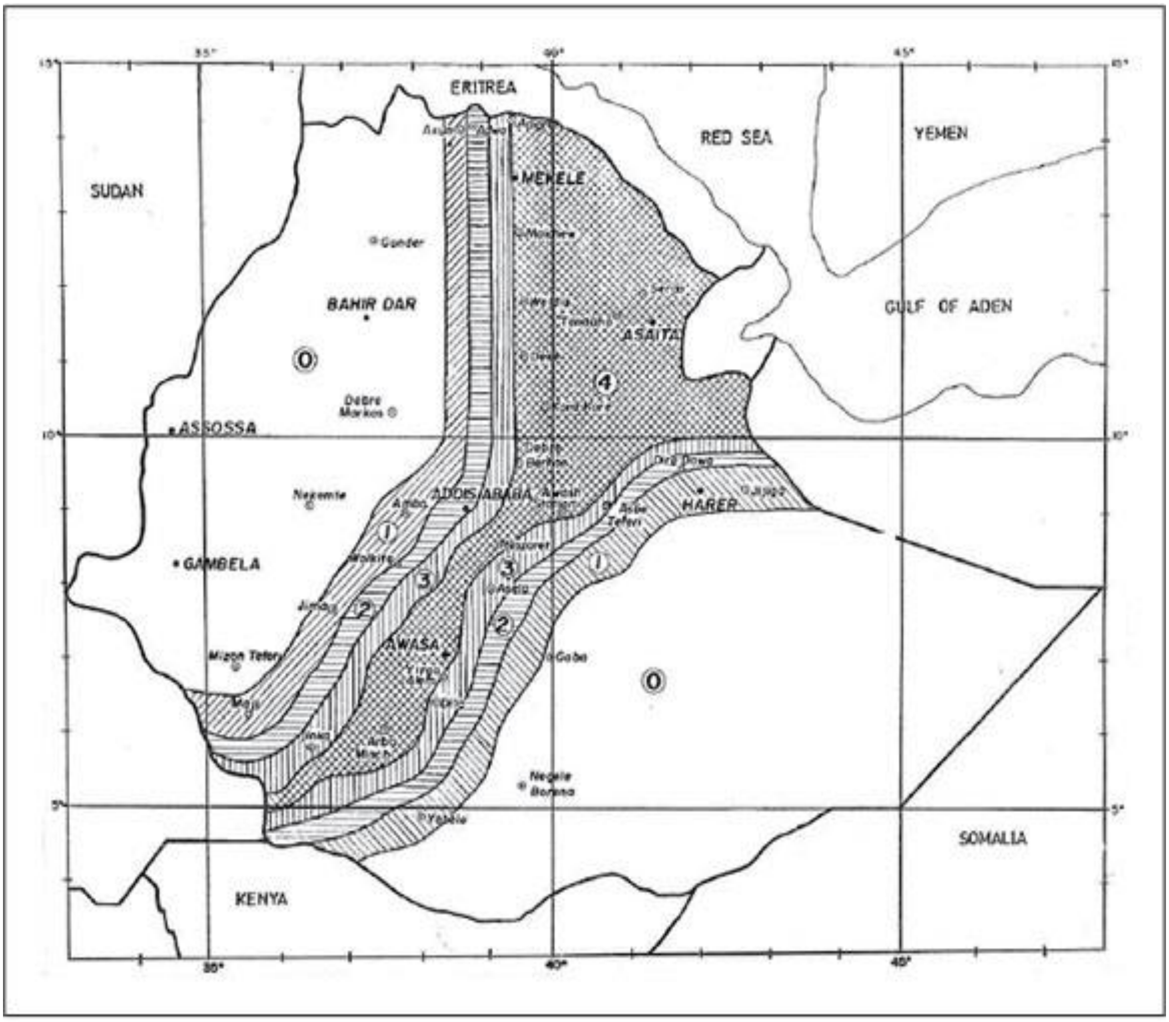

Figure 3 Seismic hazard map of Ethiopia for 100-year return period as per EBCS 8: 1995 (MWUD 1995)

Fig 1.1 Seismic Zoning Map of Ethiopia

The methodologies available so far for the evaluation of existing buildings can be divided into two categories-(i) Qualitative method (ii) Analytical method

The qualitative methods for evaluation are based on the background data of the building and its construction site available, which requires some or few documents like drawings, visual inspection report, past performance of the analogous buildings under seismic activities, and certain non-destructive test results. The analytical methods for evaluation are centered on the consideration of the ductility and capacity of buildings on the grounds of drawings which are already available. [1]

Pushover analysis is an estimated analysis method where the structure is subjected to different monotonically increasing lateral forces, with a distribution which is height-wise invariant, until the target displacement is touched. Pushover analysis comprises of a series of successive elastic analysis, superimposed to estimate a force-displacement curve of overall structure. [17]

First, a two or three dimensional model that includes bi-linear or tri-linear load-deformation figures of all the lateral force resisting elements is created and gravity loads are applied. Then, a predefined lateral load pattern that is distributed along the building height is applied. Until some members yield, the lateral forces are amplified. The structural model is modified in order to account for reduced stiffness of the yielded members and the lateral forces are increased again till additional members yield. This process is continued till a control displacement at top of the building reaches a particular level of deformation or else the structure becomes unsteady. The roof displacement is plotted with respect to the base shear so as to get the global capacity curve. [12]

Pushover analysis can be performed as force-controlled or displacement-controlled. In force-controlled pushover procedure, full load combination is applied as specified, i.e, force-controlled procedure should be used when the load is known (such as gravity loading). Also, in force-controlled pushover procedure some numerical problems that affect the accuracy of results occur since target displacement may be associated with a very small positive or even a negative lateral stiffness because of the development of mechanisms and P-delta effects.

Deformation controlled is ductile behavior and force controlled actions are brittle behavior. The following three curves Type-1, Type-2 and Type-3 are specifying the type of behavior. In case of plastic analysis methods, performance criteria are checked in terms of the deformations for ductile components and in terms of forces for brittle components.

Codes for existing buildings (FEMA 356, EN-1998-3) has given that for ductile components design deformation is less than or equal to the capacity and for brittle components the design force less than or equal to strength determined using characteristic material properties.

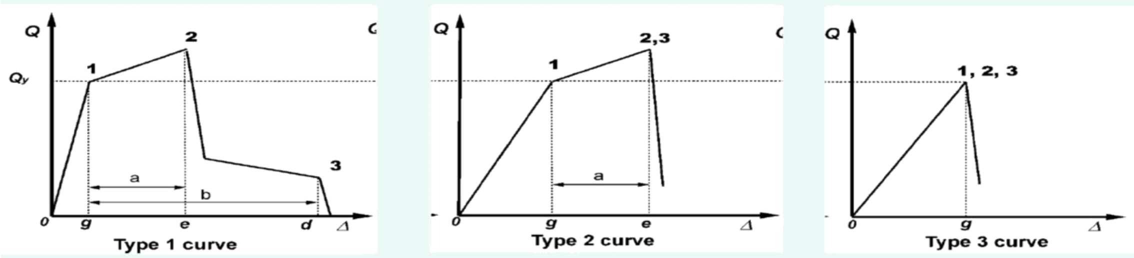

Fig 1.2 Ductile and Brittle Behaviors

Pushover analysis has been the preferred method for seismic performance evaluation of structures by the major rehabilitation guidelines and codes because it is conceptually and computationally simple. Pushover analysis allows tracing the sequence of yielding and failure on member and structural level as well as the progress of overall capacity curve of the structure.

Equivalent static method is used to seismically design most of the low and medium-rise building structures. In this method, design forces are acquired from elastic spectra that are reduced using a response modification factor. This coefficient signifies the structure’s inelastic performance and specifies hidden ductility and strength of those structures in inelastic phase.

The ratio of eventual deformation of the structure and its deformation in yielding is referred to as the ductility coefficient which expresses inelastic deformation capacity of these structures. The larger the value of this coefficient is, the higher the level of energy absorption is and the more the number of plastic joints formed are, as compared to before. Thus accurate determination of the yielding points and the ultimate displacements is very important. Certain failure criteria are used to evaluate the building's seismic demands in this study. The maximum drift of the structure without total collapse under seismic loads is called the target displacement. [11]

If the Nonlinear Static Procedure (NSP) is selected for seismic analysis of the building, a mathematical model directly incorporating the nonlinear load-deformation characteristics of individual components and elements of the building shall be subjected to monotonically increasing lateral loads representing inertia forces in an earthquake until a target displacement is exceeded. The target displacement is intended to represent the maximum displacement likely to be experienced during the design earthquake. Because the mathematical model accounts directly for effects of material inelastic response, the calculated internal forces will be reasonable approximations of those expected during the design earthquake. The relation between base shear force and lateral displacement of the control node shall be established for control node displacements ranging between zero and 150% of the target displacement, $_t$. [28]

In order to obtain performance points as well as the location of hinges in different stages, we can use the pushover curve. In this curve, the range AB being the elastic range, B to IO being the range of instant occupancy, IO to LS being the range of life safety and LS to CP being the range of collapse prevention. [17]

When a hinge touches point C on its force-displacement curve then that hinge must start to drop load. The manner in which the load is released from a hinge that has reached point C is that the pushover force or the base shear is reduced till the force in that hinge is steady with the force at point D. [17]

As the force is released, all of the elements unload as well as the displacement is decreased. After the yielded hinge touches the point D force level, the magnitude of pushover force is again amplified and the displacement starts to increase again. [17]

If all of the hinges are within the given CP limit then that structure is supposed to be safe. Though, the hinge after IO range may also be required to be retrofitted depending on the significance of structure. [17]

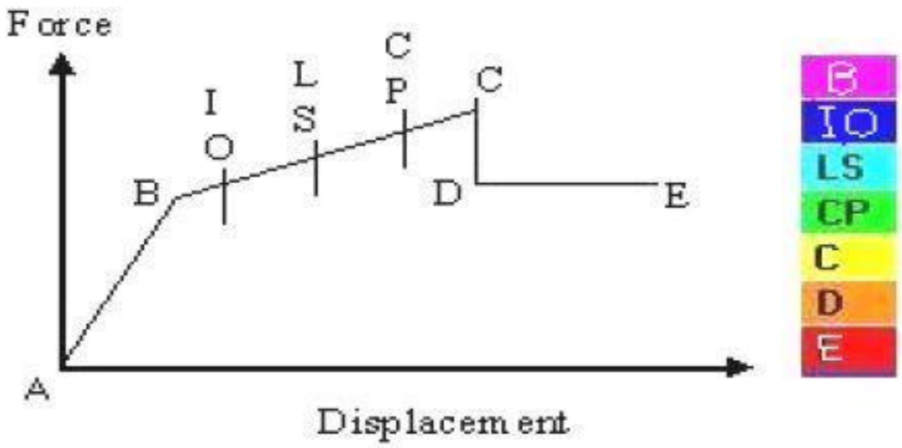

Fig. 1.3 Different stages of Plastic Hinges [17]

The basic seismic response parameters taken into consideration are- (i). Stiffness (ii). Strength (iii). Ductility.

Now, if we consider any Reinforced Concrete frame building, we can summarize the sources of weakness as:

(i). Discontinuous load path/interrupted load path/irregular load path.

(ii). Lack of deformation capability of structural members.

(iii). Quality of workmanship and materials.

1.2. (a) PROPOSED WORK AND OBJECTIVE

My research Thesis aims at doing seismic evaluation for the building located in Walyta sodo using non- linear static analysis method. Since the area is lies at highly seismic, recently the lateral effects prevailing on the structures of around walyta Sodo areas (For instance, before two years a building has been collapsed and excessive cracks are now has observed on structural elements of some Buildings).

The building is currently the most prominent building in the total walyta area. However, since it was constructed some 15 years earlier and it necessitated to evaluate the performance of this building whether it withstand or not severe earthquakes occur.

The calculations done using Equivalent Static Method reveals that the structure will invariably fail when subjected to earthquake loads. Except beams of corridors which fail in both sagging and hogging moments, all other beams were found to pass in hogging moments only. In case of columns, the ground floor columns of classrooms pass in flexural strength but the ground floor column of corridor fails in flexure. Most beams and columns were found to pass in shear. Taking the results from Equivalent Static analysis, the following objectives are formulated.

General objective:

(i) Analyze the seismic performance of the existing structure with more degree of accuracy by using Non-linear Static Analysis Method.

Specific objective:

(i). Analyze the structure in SAP-2000 in accordance to the design generated by STAAD.Pro v8i and run Pushover analysis for the limiting case of the structure to generate a pushover curve.

(ii). Find the target displacement of the structure by using Idealized Force-Displacement Curve and Displacement Coefficient Method in accordance with ASCE 41-06.

(iii). Studying the behavior of the structure when subjected to the Pushover Analysis by limiting the maximum displacement of the top node to the calculated target displacement.

1.2 (b) Problem Statement

Due to active movements of the tectonic plates the seismic activity is going to be changing from time to time. In lieu the PGA value for Ethiopia is now reached to 0.1g from 0.05g. The structures constructed by designing with the later value may be susceptible to earthquake forces and the structures some are even not constructed by considering the Earthquake perspective. This thesis is for finding those structures which are designed by no considering Earthquake forces vulnerability and finding the required retrofitting to with stand the Earthquake forces.

1.3 OUTLINE OF THE WORK

The present study deals with the non-linear static pushover analysis of a 15-year old 4-story reinforced concrete structure by the use of SAP-2000. In the process target displacement is calculated using displacement coefficient method in accordance with ASCE 41-06. The simulation of the structure analyzed in SAP-2000 was first designed in STAAD.Pro v8i considering using EBCS-2, 2013 and EBCS-8, 2013(i.e., they are similar to the Euro Code) by using C20 as concrete and yield stress is 250Mpa to be the reinforcement steel (assuming these materials to have been used 15 years ago). The structure was designed for only dead and live loads, since earthquake loads would not have been a part of the original design.

Organization of Thesis:

The thesis contains five chapters. The first chapter being the introduction, which gives a superficial insight into the work which is undertaken in the project. It gives a brief description of the field of study and the various methods which may have been used for the purpose of analysis and further calculations.

The second chapter entails a detailed review of literature pertinent to the previous works done in the field under consideration. A critical discussion of the earlier works is done. The objective and present scope of study is also outlined in this chapter.

The third chapter covers the theory and formulation which includes the details about the material used, the process of simulation of the structure, base shear calculation and pushover analysis carried out for the same. The pushover curve obtained is converted into an idealized force-displacement curve and the target displacement is calculated for both the axes using the displacement coefficient method in accordance with ASCE 41-06.

The fourth chapter contains results which were obtained post analysis. The loading diagram has been shown along with the pushover curve and inter-story drift plot. The pushover curves obtained for the target displacement limits along both the axes.

The fifth chapter lists the conclusion drawn from the work and the future scope in the area.

Chapter 2

LITERATURE REVIEW

2. Literature Review

2.1 GENERAL

M C Griffith and A V Pinto [6] have investigated the specific details of a 4-story, 3-bay reinforced concrete frame test structure with unreinforced brick masonry (URM) infill walls with attention to their weaknesses with regards to seismic loading. The concrete frame was shown to be a “weak-column strong-beam frame” which is likely to exhibit poor post yield hysteretic behavior. The building was expected to have maximum lateral deformation capacities corresponding to about 2% lateral drift. The unreinforced masonry infill walls were likely to begin cracking at much smaller lateral drifts, of the order of 0.3%, and completely lost their load carrying ability by drifts of between 1% and 2%.

Shunsuke Otani [15] studied the development of earthquake resistant design of RCC Buildings (Past and Future). The measurement of ground acceleration started in 1930’s, and the response calculation was made possible in 1940’s. Design response spectra were formulated in the late 1950’s to 1960’s. Non-linear response was introduced in seismic design in 1960’s and the capacity design concept was introduced in 1970’s for collapse safety. The damage statistics of RCC buildings in 1995 Kobe disaster demonstrated the improvement of building performance with the development of design methodology. Buildings designed and constructed using outdated methodology should be upgraded. Performance basis engineering should be emphasized, especially for the protection of building functions following frequent earthquakes.

Ciro Faella, Enzo Martinelli, Emidio Nigro [4] proposed an assessment procedure in terms of displacement capacity and demand. The sample application of the proposed procedure to a typical building emphasized how easy and quick can be its application. As a brief parametrical investigation, the influence of subsoil stiffness on the seismic vulnerability of the building was analyzed pointing out that vulnerability was much larger as subsoil was less stiff. A rational design procedure for choosing the retrofitting system was proposed with the aim of determining the key mechanical characteristics of a bracing system working in parallel with the existing structure for complying the safety requirement provided by Euro code 8 – Part 3 entirely devoted to existing structures. In the proposed design procedure, according to a displacement-based-approach, the strengthening substructure was designed in terms of lateral stiffness, because displacement demand is strictly controlled by the displacement capacity of the existing structure. For this reason, usual force-based design procedures suitable for new structures, in which displacement capacity is only imposed by the new structure itself, are not directly applicable for bracing system utilized for retrofitting existing structures.

Oğuz, Sermin [21] ascertained the effects and the accuracy of invariant lateral load patterns utilized in pushover analysis to predict the behavior imposed on the structure due to randomly selected individual ground motions causing elastic deformation by studying various levels of nonlinear response. For this purpose, pushover analyses using various invariant lateral load patterns and Modal Pushover Analysis were performed on reinforced concrete and steel moment resisting frames covering a broad range of fundamental periods. The accuracy of approximate procedures utilized to estimate target displacement was also studied on frame structures. Pushover analyses were performed by both DRAIN-2DX and SAP2000. The primary observations from the study showed that the accuracy of the pushover results depended strongly on the load path, the characteristics of the ground motion and the properties of the structure.

Durgesh C. Rai [17] gave the guidelines for seismic evaluation and strengthening of buildings. This document was developed as part of project entitled ―Review of Building Codes and Preparation of Commentary and Handbooks, awarded to Indian Institute of Technology Kanpur by the Gujarat State Disaster Management Authority (GSDMA), Gandhinagar through World Bank finances. This document was particularly concerned with the seismic evaluation and strengthening of existing buildings and it was intended to be used as a guide.

G E Thermou and A S Elnashai [23] made a global assessment of the effect of repair methods on ductility, strength and stiffness, the three most important seismic response parameters, to assist researchers and practitioners in decision-making to satisfy their respective intervention aims. Also the term ‘rehabilitation’ was used as a comprehensive term to include all types of retrofitting, repair and strengthening that leads to reduced earthquake vulnerability. The term ‘repair’ was defined as reinstatement of the original characteristics of a damaged section or element and was confined to dealing with the as-built system. The term ‘strengthening’ was defined as intervention that lead to enhancement of one or more seismic response parameters (ductility, strength, stiffness, etc.), depending on the desired performance.

A.Kadid and A. Boumrkik [8] proposed use of Pushover Analysis as a viable method to assess damage vulnerability of a building designed according to Algerian code. Pushover analysis was a series of incremental static analysis carried out to develop a capacity curve for the building. Based on the capacity curve, a target displacement which was an estimate of the displacement that the design earthquake would produce on the building was determined. The extent of damage experienced by the structure at this target displacement is considered representative of the damage experienced by the building when subjected to design level ground shaking. Since the behavior of reinforced concrete structures might be highly inelastic under seismic loads, the global inelastic performance of RC structures would be dominated by plastic yielding effects and consequently the accuracy of the pushover analysis would be influenced by the ability of the analytical models to capture these effects.

R.K. Goel [7] evaluated the nonlinear static procedures specified in the FEMA-356, ASCE/SEI 41-06, ATC-40, and FEMA-440 documents for seismic analysis and evaluation of building structures using strong-motion records of RC buildings. The maximum roof displacement predicted from the nonlinear static procedure was compared with the value derived directly from recorded motions for this purpose. It was shown that: (i) the nonlinear static procedures either overestimates or underestimates the peak roof displacement for several of the buildings considered in the investigation; (ii) the ASCE/SEI 41-06 Coefficient Method (CM), which was based on recent improvements to the FEMA-356 Coefficient Method suggested in the FEMA-440 document, does not necessarily provide better estimate of the roof displacement; and (iii) the improved FEMA-440 Capacity Spectrum Method (CSM) provided better estimates of the roof displacement compared to the ATC-40 CSM.

Saptadip Sarkar [19] studied the Design of Earthquake resistant multi stories RCC building on a sloping ground that involves the analysis of simple 2-D frames of different floor heights and varying number of bays using a software tool named STAAD Pro. Using the analysis results various graphs were drawn between the maximum compressive stress, maximum bending moment, maximum shear force, maximum tensile force and maximum axial force being developed for the frames on plane ground and sloping ground. The graphs were used to draw comparisons between the two cases and the detailed study of Short Column Effect failure. In addition to that, the feasibility of the software tool to be used was also checked and the detailed study of seismology was undertaken.

Siamak Sattar and Abbie B. Liel [20] quantified the effect of the presence and configuration of masonry infill walls on seismic collapse risk. Infill panels are modeled by two nonlinear strut elements, which have compressive strength only. Nonlinear models of the frame-wall system were subjected to incremental dynamic analysis in order to assess seismic performance. There was an increase observed in initial strength, stiffness, and energy dissipation of the infilled frame, when compared to the bare frame, even after the wall's brittle failure modes. Dynamic analysis results indicated that fully-infilled frame had the lowest collapse risk and the bare frames were found to be the most vulnerable to earthquake-induced collapse. The better collapse performance of fully-infilled frames was associated with the larger strength and energy dissipation of the system, associated with the added walls.

Benyamin Monavari, Ali Massumi & Alireza Kazem [11] used nonlinear static analysis and five locals and overall yields and failure criteria to estimate seismic demands of buildings. The failure is directed towards losing structure's performance during the earthquake or subsequent effects. Because of the consequent excitations of an earthquake or lateral imposed loads on a structure, the stiffness of some elements of structure reduced and the structure started to fail and lose its performance; although failure happened either in small parts of structure or at the whole. In this study thirteen reinforced concrete (RC) frame buildings with 2, 3, 4, 5, 6, 7, 8, 9, 10, 11, 12, 16 and 20 stories, having 3 and 4 bays were designed using seismic force levels obtained from the Iranian Seismic Code 2005 and proportioned using the ACI318-99 Building Code and then were modeled by IDARC. Pushover analysis with increasing triangular loading was used.

Haroon Rasheed Tamboli & Umesh N. Karadi [22] performed seismic analysis using Equivalent Lateral Force Method for different reinforced concrete (RC) frame building models that included bare frame, infilled frame and open first story frame. In modeling of the masonry infill panels the Equivalent diagonal Strut method was used and the software ETABS was used for the analysis of all the frame models. Infilled frames should be preferred in seismic regions than the open first story frame, because the story drift of first story of open first story frame is very large than the upper stories, which might probably cause the collapse of structure. The infill wall increases the strength and stiffness of the structure. The seismic analysis of RC (Bare frame) structure lead to under estimation of base shear. Therefore other response quantities such as time period, natural frequency, and story drift were not significant. The underestimation of base shear might lead to the collapse of structure during earthquake shaking.

Narender Bodige, Pradeep Kumar Ramancharla [3] modeled a 1 x 1 bay 2D four storied building using AEM (applied element method). AEM is a discrete method in which the elements are connected by pair of normal and shear springs which are distributed around the elements edges and each pair of springs totally represents stresses and deformation and plastic hinges location are formed automatically. Gravity loads and laterals loads as per IS 1893-2002 were applied on the structure and designed using IS 456 and IS 13920. Displacement control pushover analysis was carried out in both cases and the pushover curves were compared. As an observation it was found that AEM gave good representation capacity curve. From the case studies it was found that capacity of the building significantly increased when ductile detailing was adopted. Also, it was found that effect on concrete grade and steel were not highly significant.

Humar and Wright (1977) studied the dynamic behavior of multi-storied steel frame buildings with setbacks. The observations made based on a detailed parametric study are as follows. The fundamental period decreased by 35% for a setback of 90% (i.e., tower occupying 10% of the base area). The higher mode vibration of setback buildings made substantial contribution to their seismic response; these contributions increased with the slenderness of the tower. The contribution of the higher modes increased to 40% for a setback of 90%. For very slender towers the transition region between the tower and the base was, in some cases, subjected to very large storey shears. This increase in shear force was found to be as high as 300% to 400% for a setback of 90%. Storey drift ratios and storey shears for tower portions of setback buildings were substantially larger than for building without setbacks. For the tower portion, the increase in inter-storey drift was found to be four times compared to that of a regular structure. This increase was influenced by the extent of the setback. It was also observed that beam ductility demand in the tower portion showed a large increase with increase in the slenderness of the tower. The column ductility demands in the tower portion also showed a similar trend.

Shahrooz and Moehle (1990) studied the effects of setbacks on the earthquake response of multi-storied buildings. In an effort to improve design methods for setback structures, an experimental and analytical study was undertaken. A six-storey moment- resisting reinforced concrete space frame with 50% setback in one direction at mid-height was selected. The analytical study focused on the test structure. The displacement profiles were relatively smooth over the height. Relatively large inter-storey drifts at the tower- base junction were accompanied by a moderate increase in damage at that level. Overall, the predominance of the fundamental mode on the global translational response in the direction parallel to the setback was clear from the displacement and inertia force profiles. The distribution of lateral forces was almost always similar to the distribution specified by the UBC code; no significant peculiarities in dynamic response were detected. To investigate further, an analytical study was also carried out on six generic reinforced concrete setback frames.

Wood (1992) investigated the seismic behavior of reinforced concrete frames with steps and setbacks. Two small-scale reinforced concrete 9-storeyed test framed structures (one-with steps and the other with setbacks) were constructed and subjected to simulated ground motion. The displacement, acceleration and the shear force responses of these frames were compared with those of seven previously tested regular frames. The setback structure comprises two-story base with seven additional story's in the tower portions. The stepped structure includes a three story tower, a three story middle section and a three storey base. The displacement and shear force responses of these two frames were governed primarily by the first mode. Acceleration response at all levels exhibited the contribution of higher modes. The mode shapes for both the frames indicated kinks at the step locations. However, distributions of maximum storey shear were well represented by the equivalent lateral force distributions for all frames as given in UBC for regular frames. The differences between the linear dynamic analyses of regular, stepped and setback frames were not significant.

Ghobarah A. et al., (1997) the control of inter story drift can also be considered as a means to provide uniform ductility over the stories of the building. A story drift may result in the occurrence of a weak story that may cause catastrophic building collapse in a seismic event. Uniform story ductility over all stories for a building is usually desired in seismic design.

Foley CM. (2002) a review of current state-of-the-art seismic performance-based design procedures and presented the vision for the development of PBD optimization. It is recognized that there is a pressing need for developing optimized PBD procedures for seismic engineering of structures.

R. Hasan and L. Xu, D.E. Grierson (2002) conducted a simple computer-based push- over analysis technique for performance-based design of building frameworks subject to earthquake loading. And found that rigidity-factor for elastic analysis of semi-rigid frames, and the stiffness properties for semi-rigid analysis are directly adopted for push- over analysis.

B. AKBAS.*et.al*.(2003) conducted a push over analysis on steel frames to estimate the seismic demands at different performance levels, which requires the consideration of inelastic behavior of the structure.

X.-K. Zou et al., (2005) presented an effective technique that incorporates Pushover Analysis together with numerical optimization procedures to automate the Pushover drift performance design of reinforced concrete buildings. PBD using nonlinear pushover analysis, which generally involves tedious computational effort, is highly iterative process needed to meet code requirements.

Oğuz, Sermin (2005) Ascertained the effects and the accuracy of invariant lateral load patterns Utilized in pushover analysis to predict the behavior imposed on the structure due to randomly Selected individual ground motions causing elastic deformation by studying various levels of Nonlinear response. For this purpose, pushover analyses using various invariant lateral load Patterns and Modal Pushover Analysis were performed on reinforced concrete and steel moment resisting frames covering a broad range of fundamental periods. The accuracy of approximate Procedures utilized to estimate target displacement was also studied on frame structures. Pushover analyses were performed by both DRAIN-2DX and SAP2000. The primary observations from the study showed that the accuracy of the pushover results depended strongly On the load path, the characteristics of the ground motion and the properties of the structure.

Mehmet et al., (2006), explained that due to its simplicity of Push over analysis, the structural engineering profession has been using the nonlinear static procedure or pushover analysis. Pushover analysis is carried out for different nonlinear hinge properties available in some programs based on the FEMA-356 and ATC-40 guidelines and he pointed out that Plastic hinge length (Lp) has considerable effects on the displacement capacity of the frames. The orientation and the axial load level of the columns cannot be taken into account properly by the default-hinge properties (Programme Default).

Shuraim et al., (2007) summarized the nonlinear static analytical procedure (Pushover) as introduced by ATC-40 has been utilized for the evaluation of existing design of a building, in order to examine its applicability. He conducted nonlinear pushover analysis shows that the frame is capable of withstanding the pre-assumed seismic force with some significant yielding at all beams and columns.

Girgin. et.,(2007),Pushover analysis has been the preferred method for seismic performance evaluation of structures by the major rehabilitation guidelines and codes because it is computationally and conceptually simple. Pushover analysis allows tracing the sequence of yielding and failure on member and structural level as well as the progress of overall capacity curve of the structure.

A. Shuraim et al., (2007) the nonlinear static analytical procedure (Pushover) as introduced by ATC-40 has been utilized for the evaluation of existing design of a new reinforced concrete frame. Potential structural deficiencies in reinforced concrete frame, when subjected to a moderate seismic loading, were estimated by the pushover approaches. In this method the design was evaluated by redesigning under selected seismic combination in order to show which members would require additional reinforcement. Most columns required significant additional reinforcement, indicating their vulnerability when subjected to seismic forces. The nonlinear pushover procedure shows that the frame is capable of withstanding the presumed seismic force with some significant yielding at all beams and one column.

Athanassiadou (2008) analyzed two ten-storied two-dimensional plane stepped frames and one ten-storied regular frame designed, as per Euro code 8 (2004) for the high and medium ductility classes. This research validates the design methodology requiring linear dynamic analysis recommended in Euro code 8 for irregular buildings. The stepped buildings, designed to Euro code 8 (2004) were found to behave satisfactorily under the design basis earthquake and also under the maximum considered earthquake (involving ground motion twice as strong as the design basis earthquake). Inter-storey drift ratios of irregular frames were found to remain quite low even in the case of the „collapse prevention" earthquake. This fact, combined with the limited plastic hinge formation in columns, exclude the possibility of formation of a collapse mechanism at the neighborhood of the irregularities. Plastic hinge formation in columns is seen to be very limited during the design basis earthquake, taking place only at locations not prohibited by the code, i.e. at the building base and top. It has been concluded that the capacity design procedure provided by Euro code 8 is completely successful and can be characterized by conservatism, mainly in the case of the design of high-ductility columns. The over-strength of the irregular frames is found to be similar to that of the regular ones, with the over-strength ratio values being 1.50 to 2.00 for medium – high ductility levels. The author presented the results of pushover analysis using „uniform‟ load pattern as well as a „modal‟ load pattern that account the results of multimodal elastic analysis.

Karavasilis et. al. (2008) presented a parametric study of the inelastic seismic response of plane steel moment resisting frames with steps and setbacks. A family of 120 such frames, designed according to the European seismic and structural codes, were subjected to 30 earthquake ground motions, scaled to different intensities. The main findings of this paper are as follows. Inelastic deformation and geometrical configuration play an important role on the height-wise distribution of deformation demands. In general, the maximum deformation demands are concentrated in the tower-base junction in the case of setback frame and in all the step locations in the case of stepped frames. This concentration of forces at the locations of height discontinuity, however, is not observed in the elastic range of the seismic response.

A.Kadid and A. Boumrkik (2008), proposed use of Pushover Analysis as a viable method to assess damage vulnerability of a building designed according to Algerian code. Pushover analysis was a Series of incremental static analysis carried out to develop a capacity curve for the building. Based on capacity curve, a target displacement which was an estimate of the displacement that the design earthquake would produce on the building was determined. The extent of damage Experienced by the structure at this target displacement is considered representative of the Damage experienced by the building when subjected to design level ground shaking. Since the behavior of reinforced concrete structures might be highly inelastic under seismic loads, the global inelastic performance of RC structures would be dominated by plastic yielding effects and consequently the accuracy of the pushover analysis would be influenced by the ability of the Analytical models to capture these effects.

Kala.Pet. al. (2010), conducted study on steel water tanks designed as per recent and past I. S codes and they found Compression members are more critical than tension members. And he pointed out that, in Limit state method the partial safety factors on load and material have been derived using the probability concept which is more rational and realistic

P.Poluraju and P.V.S.N.Rao (2011), has studied the behavior of framed building by conducting Push over Analysis, most of buildings collapsed were found deficient to meet out the requirements of the present day codes.

Then G+3 building was modeled and analyzed, results obtained from the study shows that properly designed frame will perform well under seismic loads.

Haroon Rasheed Tamboli & Umesh N. Karadi (2012), performed seismic analysis using Equivalent Lateral Force Method for different reinforced concrete (RC) frame building models that included bare frame, in filled frame and open first story frame. In modeling of the masonry Infill panels the Equivalent diagonal Strut method was used and the software ETABS was used for the analysis of all the frame models. In filled frames should be preferred in seismic regions than the open first story frame, because the story drift of first story of open first story frame is Very large than the upper stories, which might probably cause the collapse of structure. The infill Wall increases the strength and stiffness of the structure. The seismic analysis of RC (Bare frame) structure lead to under estimation of base shear. Therefore other response quantities such as time period, natural frequency, and story drift were not significant. The underestimation of base shear might lead to the collapse of structure during earthquake shaking.

Narender Bodige, Pradeep Kumar Ramancharla (2012), modeled a 1 x 1 bay 2D four storied building using AEM (applied element method). AEM is a discrete method in which the elements are connected by pair of normal and shear springs which are distributed around the elements edges and each pair of springs totally represents stresses and deformation and plastic hinges location are formed automatically. Gravity loads and laterals loads as per IS 1893-2002 were applied on the structure and designed using IS456 and IS 13920. Displacement control pushover analysis was carried out in both cases and the pushover curves were compared. As an observation it was found that AEM gave good representation capacity curve. From the case studies it was found that capacity of the building significantly increased when ductile detailing was adopted. Also, it was found that effect on concrete grade and steel were not highly significant.

2.2 SUMMARY OF REVIEW

Pushover analysis yields insight into elastic and inelastic response of structures under earthquakes provided that adequate modeling of structure, careful selection of lateral load pattern and careful interpretation of results are performed. However, pushover analysis is more appropriate for low to mid-rise buildings with dominant fundamental mode response. For special and high-rise buildings, pushover analysis should be complemented with other evaluation procedures since higher modes could certainly affect the response.

2.3STUDY AREA

Seismic Engineering is a sub discipline of the broader category of Structural engineering. Its main objectives therefore are-

-To understand interaction of structures with the shaky ground.

-To foresee the consequences of possible earthquakes.

-To design, construct and maintain structures to perform at earthquake exposure up to the expectations and in compliance with building codes.

The methodologies available so far for the evaluation of existing buildings can be divided into two categories-(i) Qualitative method (ii) Analytical method.

In the same realm, seismic analysis is a subset of structural analysis and is the calculation of the response of a structure to earthquakes. It is part of the process of structural design, earthquake engineering or structural assessment and retrofit in regions where earthquakes are prevalent. Structural analysis methods can be divided into the following categories-

2.3.1Structural analysis methods

Elastic Analysis

$$\left.\begin{matrix} Lateral\ Force\ Method \\ Modal\ Response\ spectrum\ Analysis(Spectral\ Analysis) \end{matrix}\right\} \textbf{Conventional Design}$$

$$\left.\begin{matrix} Modal\ Time\ History\ Analysis \\ Linear\ Dynamic\ Analysis \\ \boldsymbol{Inelastic\ Analysis}: i)Non - Linear\ static\ Analysis(Push\ over\ ) \\ ii)\ Non\ linear\ Dynamic\ Analysis \end{matrix}\right\} \boldsymbol{Advanced\ Design}$$

2.3.1.1Equivalent static analysis

This approach defines a series of forces acting on a building to represent the effect of earthquake ground motion, typically defined by a seismic design response spectrum. It assumes that the building responds in its fundamental mode. For this to be true, the building must be low-rise and must not twist significantly when the ground moves. The response is read from a design response spectrum, given the natural frequency of the building (either calculated or defined by the building code). The applicability of this method is extended in many building codes by applying factors to account for higher buildings with some higher modes, and for low levels of twisting. To account for effects due to "yielding" of the structure, many codes apply modification factors that reduce the design forces (e.g. force reduction factors).

2.3.1.2Response spectrum analysis

This approach permits the multiple modes of response of a building to be taken into account (in the frequency domain). This is required in many building codes for all except for very simple or very complex structures. The response of a structure can be defined as a combination of many special shapes (modes) that in a vibrating string correspond to the "harmonics". Computer analysis can be used to determine these modes for a structure. For each mode, a response is read from the design spectrum, based on the modal frequency and the modal mass, and they are then combined to provide an estimate of the total response of the structure. In this we have to calculate the magnitude of forces in all directions i.e. X, Y & Z and then see the effects on the building.. Combination methods include the following:

Absolute - peak values are added together

Square root of the sum of the squares (SRSS)

complete quadratic combination (CQC) - a method that is an improvement on SRSS for closely spaced modes

The result of a response spectrum analysis using the response spectrum from a ground motion is typically different from that which would be calculated directly from a linear dynamic analysis using that ground motion directly, since phase information is lost in the process of generating the response spectrum.

In cases where structures are either too irregular, too tall or of significance to a community in disaster response, the response spectrum approach is no longer appropriate, and more complex analysis is often required, such as non-linear static analysis or dynamic analysis.

2.3.1.3Linear dynamic analysis

Static procedures are appropriate when higher mode effects are not significant. This is generally true for short, regular buildings. Therefore, for tall buildings, buildings with torsional irregularities, or non-orthogonal systems, a dynamic procedure is required. In the linear dynamic procedure, the building is modelled as a multi-degree-of-freedom (MDOF) system with a linear elastic stiffness matrix and an equivalent viscous damping matrix.

The seismic input is modeled using either modal spectral analysis or time history analysis but in both cases, the corresponding internal forces and displacements are determined using linear elastic analysis. The advantage of these linear dynamic procedures with respect to linear static procedures is that higher modes can be considered. However, they are based on linear elastic response and hence the applicability decreases with increasing nonlinear behavior, which is approximated by global force reduction factors.

In linear dynamic analysis, the response of the structure to ground motion is calculated in the time domain, and all phase information is therefore maintained. Only linear properties are assumed. The analytical method can use modal decomposition as a means of reducing the degrees of freedom in the analysis.

2.3.1.4Nonlinear static analysis

In general, linear procedures are applicable when the structure is expected to remain nearly elastic for the level of ground motion or when the design results in nearly uniform distribution of nonlinear response throughout the structure. As the performance objective of the structure implies greater inelastic demands, the uncertainty with linear procedures increases to a point that requires a high level of conservatism in demand assumptions and acceptability criteria to avoid unintended performance. Therefore, procedures incorporating inelastic analysis can reduce the uncertainty and conservatism.

This approach is also known as "pushover" analysis. A pattern of forces is applied to a structural model that includes non-linear properties (such as steel yield), and the total force is plotted against a reference displacement to define a capacity curve. This can then be combined with a demand curve (typically in the form of an acceleration-displacement response spectrum (ADRS)). This essentially reduces the problem to a single degree of freedom (SDOF) system.

Nonlinear static procedures use equivalent SDOF structural models and represent seismic ground motion with response spectra. Story drifts and component actions are related subsequently to the global demand parameter by the pushover or capacity curves that are the basis of the non-linear static procedures.

2.3.1.5Nonlinear dynamic analysis

Nonlinear dynamic analysis utilizes the combination of ground motion records with a detailed structural model, therefore is capable of producing results with relatively low uncertainty. In nonlinear dynamic analyses, the detailed structural model subjected to a ground-motion record produces estimates of component deformations for each degree of freedom in the model and the modal responses are combined using schemes such as the square-root-sum-of-squares.

In non-linear dynamic analysis, the non-linear properties of the structure are considered as part of a time domain analysis. This approach is the most rigorous, and is required by some building codes for buildings of unusual configuration or of special importance. However, the calculated response can be very sensitive to the characteristics of the individual ground motion used as seismic input; therefore, several analyses are required using different ground motion records to achieve a reliable estimation of the probabilistic distribution of structural response. Since the properties of the seismic response depend on the intensity, or severity, of the seismic shaking, a comprehensive assessment calls for numerous nonlinear dynamic analyses at various levels of intensity to represent different possible earthquake scenarios. This has led to the emergence of methods like the Incremental Dynamic Analysis.

In this study we have used “Pushover Analysis” for assessment of the considered four-story RC structure. Pushover Analysis is essentially the extension of the “lateral force procedure” of static analysis into non-linear regime. It is carried out under constant gravity loads and monotonically increasing lateral loading applied on the masses of the structural model. [5]. Assume that the response is governed by single mode of vibration and that is constant during the analysis.

Distribution of lateral forces (applied at storey masses). i) Modal-usually first mode i.e, inverted triangle ii) Uniform- lateral forces proportional to story masses.

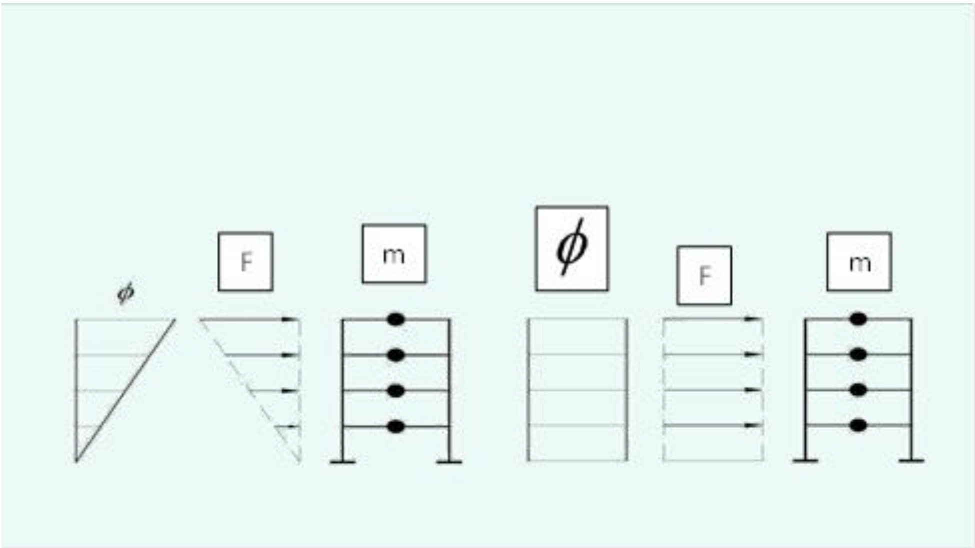

Fig.2.1Distribution of lateral forces in model and uniform

Non Linear static analysis applicable to low rise regular buildings, whereas the response is dominated by the fundamental mode of vibration. This method represents a direct evaluation of overall structural response not only on element by element basis. Also allows evaluation of inelastic deformations, this is the most relevant response quantity in the case of inelastic response.

A pattern of forces is applied to a structural model that includes non-linear properties (such as steel yield), and the total force is plotted against a reference displacement to define a capacity curve. This loading is meant to simulate inertia forces due to only the horizontal component of the seismic action, neglecting the vertical component altogether. While the applied lateral forces increase in the course of analysis, the engineer can follow the gradual emergence of plastic hinges, the evolution of plastic mechanism and damage, as a function of the magnitude of the imposed lateral loads and of the resulting displacements. [5]

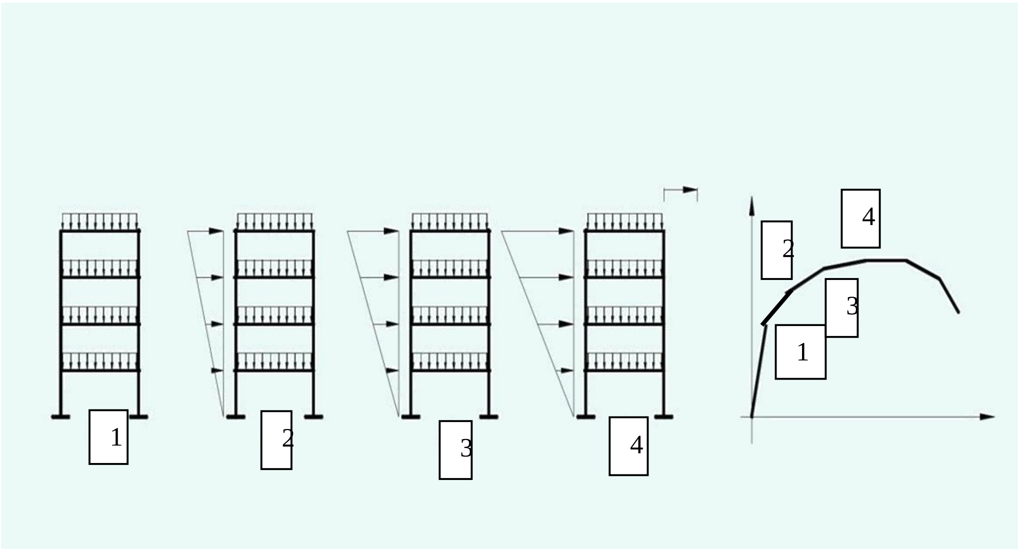

Fig 2.2 Step wise increment of lateral forces

Unlike linear or non-linear dynamic analysis, which both give directly all peak seismic demands under a given earthquake, a pushover analysis per se gives only the capacity curve. The demand has to be estimated separately. This is normally done in terms of the maximum displacement induced by the earthquake, either to the equivalent SDOF system or at the control node of the full structure. This is called “target displacement”. [5]

The demands at the local level (inelastic deformations and forces) due to the horizontal component of the seismic action in the direction of the pushover analysis are those corresponding to the “target displacement”. It is required to carry out the pushover until a terminal point at 1.5 times the “target displacement”. [5]Target displacement can be determined by any of the following methods: (i) Capacity Spectrum Method (ii) Displacement Coefficient Method (iii) N2 Method.

Chapter 3

THEORY AND FORMULATION

3. Theory and Formulation

3.1 Non-linear static Pushover Analysis-The Concept:

The pushover analysis of a structure is a static non-linear analysis under permanent vertical loads and gradually increasing lateral loads. A plot of total base shear versus top displacement in a structure is obtained by this analysis that would indicate a premature failure or weakness. All the beams and columns which reach yield or have experienced crushing and even fracture are identified. A plot of total base shear versus inter-story drift is also obtained. A pushover analysis is performed by subjecting a structure to a monotonically increasing pattern of lateral loads, that shows the inertial forces which would be experienced by the structure when subjected to ground motion. Under incrementally increasing loads many structural elements may yield sequentially. Therefore, at each event, the structure experiences a decrease in stiffness. Using a non- linear static pushover analysis, a representative non-linear force displacement relationship can be obtained.

3.1.1 Background:

Nonlinear static analysis, or pushover analysis, has been advanced over the past twenty years and has now become the most preferred analysis technique for design and seismic performance estimation purposes as this technique is comparatively simple and considers post- elastic performance. However, this technique includes certain approximations and simplifications due to which some extent of variation is always probable to exist in the seismic demand prediction of pushover analysis. [13]

Though, pushover analysis is known to capture vital structural response characteristics when the structure is under seismic action, however the reliability and the accuracy of pushover analysis in estimating global and local seismic demands for all of the structures

have been a topic of discussion and enhanced in pushover procedures have been suggested to overcome certain limitations of traditional pushover techniques. However, the improved techniques are mostly computationally hard and theoretically complex therefore use of such techniques are impractical in engineering profession and codes. As traditional pushover analysis is used widely for the design and seismic performance estimation purposes, therefore its weaknesses, limitations and predictions accuracy in routine application must be identified by studying all the factors that the pushover prediction. That is, the applicability of pushover analysis for predicting seismic demands must be investigated for low-rise, mid-rise and high-rise structures by recognizing certain issues like modeling nonlinear member performance, computational scheme of the technique, efficiency of invariant lateral load patterns in demonstrating higher mode effects, variations in the estimations of different lateral load patterns used in traditional pushover analysis and precise estimation of target displacement where seismic demand prediction of pushover technique is executed.

3.1.2 Necessity of Non-linear static Pushover Analysis:

Since the Building (structure under consideration) was constructed more than 15years ago, it may be vulnerable to seismic excitation. Hence to estimate the performance of the structure a Pushover analysis for the structure has been carried out. If the structure shows signs of failure then suitable retrofit measures may also be suggested.

3.1.3 Limitations of Pushover Analysis:

Although pushover analysis has certain advantages in comparison to elastic analysis techniques, underlying various assumptions, the accuracy of pushover predictions and the restrictions of current pushover procedures must be recognized. The estimation of target displacement, selection of the lateral load patterns and identification of failure mechanisms due to higher modes of vibration are vital issues that have an effect on the accuracy of pushover result. Target displacement is global displacement likely in a design earthquake. [9].In pushover analysis, target displacement for a multi degree of freedom (MDOF) system is generally estimated similar to the displacement demand for corresponding equivalent single degree of freedom (SDOF) system. The fundamental properties of an equivalent SDOF system are gotten from a shape vector that represents the deflected shape of MDOF system.

Most researchers recommend using normalized displacement profile at target displacement level as a shape vector, but since this displacement is not known before hand, an interaction is need. Therefore, by most of the approaches, a fixed shape vector, elastic first mode, is utilized for simplicity without regarding higher modes. The target displacement is found by the displacement at mass of the structure. [9]

The accurate estimation of the target displacement associated with particular performance objective, has an effect on accuracy of the seismic demand predictions of pushover analysis. Furthermore, hysteretic characteristics of MDOF must be incorporated into the equivalent SDOF model, in case displacement demand is affected from stiffness degradation or pinching, strength deterioration, P-Δ effects. Foundation uplift, torsional effects as well as semi-rigid diaphragms may also affect target displacement. [9]

However, in pushover analysis, usually an invariant lateral load pattern is utilized that the distribution of the inertia forces is assumed to be not changing during earthquake and deformed configuration of the structure under the action of invariant lateral load pattern is likely to be similar to that which is experienced in the design earthquake. As response of the structure, therefore the capacity curve is highly sensitive to the lateral load distribution selected choice of lateral load pattern is more critical as compared to the accurate estimation of the target displacement. [10]

The invariant load patterns cannot explain the redistribution of inertia forces because of progressive yielding and resulting variations in dynamic properties of structure. Also, fixed load patterns have inadequate capability to foretell higher mode effects in post-elastic range. These restrictions have led many researchers to suggest adaptive load patterns that consider the variations in inertia forces corresponding to the level of inelasticity. The basic approach of this technique is to restructure the lateral load shape with the degree of inelastic deformations. Although better predictions have been found from adaptive load patterns, they make pushover analysis computationally hard and theoretically complicated.

The measure of improvement has been a topic of discussion that simple invariant load patterns are preferred widely at the expense of accuracy. [14] We have used an invariant triangular loading pattern here.

Response characteristics that can be obtained from the pushover analysis are summarized as follows:

a) Estimates of force and displacement capacities of the structure. Sequence of the member yielding and the progress of the overall capacity curve.

b) Estimates of force (axial, shear and moment) demands on potentially brittle elements and deformation demands on ductile elements.

c) Estimates of global displacement demand, corresponding inter-storey drifts and damages on structural and non-structural elements expected under the 20 earthquake ground motion considered.

d) Sequences of the failure of elements and the consequent effect on the overall structural stability.

e) Identification of the critical regions, when the inelastic deformations are expected to be high and identification of strength irregularities (in plan or in elevation) of the building.

Pushover analysis delivers all these benefits for an additional computational effort (modeling nonlinearity and change in analysis algorithm) over the linear static analysis. Step by step procedure of pushover analysis is discussed next.

A displacement-controlled pushover analysis is basically composed of the following steps:

1. A two or three dimensional model that represents the overall structural behavior is created.

2. Bilinear or tri-linear load-deformation diagrams of all important members that affect lateral response are defined.

3. Gravity loads composed of dead loads and a specified portion of live loads are applied to the structural model initially.

4. A pre -defined lateral load pattern which is distributed along the building height is then applied.

5. Lateral loads are increased until some member(s) yield under the combined effects of gravity and lateral loads.

6. Base shear and roof displacement are recorded at first yielding.

7. The structural model is modified to account for the reduced stiffness of yielded member(s).

8. Gravity loads are removed and a new lateral load increment is applied to the modified structural model such that additional member(s) yield. Note that a separate analysis with zero initial conditions is performed on modified structural model under each incremental lateral load. Thus, member forces at the end of an incremental lateral load analysis are obtained by adding the forces from the current analysis to the sum of those from the previous increments. In other words, the results of each incremental lateral load analysis are superimposed.

9. Similarly, the lateral load increment and the roof displacement increment are added to the corresponding previous total values to obtain the accumulated values of the base shear and the roof displacement.

10. Steps 7, 8 and 9 are repeated until the roof displacement reaches a certain level of deformation or the structure becomes unstable.

11. The roof displacement is plotted with the base shear to get the global capacity (pushover) Curve of the structure (Figure below shown).

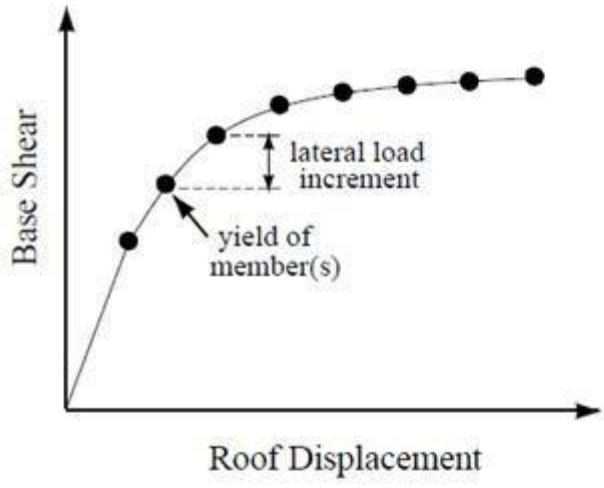

Fig 3.1: Global Capacity (Pushover) Curve of Structure

Use of Pushover Results

Pushover analysis has been the preferred method for seismic performance evaluation of structures by the major rehabilitation guidelines and codes because it is computationally and conceptually simple. Pushover analysis allows tracing the sequence of yielding and failure on member and structural level as well as the progress of overall capacity curve of the structure.

The expectation from pushover analysis is to estimate critical response parameters imposed on structural system and its components as close as possible to those predicted by nonlinear dynamic analysis. Pushover analysis provides information on many response characteristics that can’t be obtained from an elastic static or elastic dynamic analysis.

These are

- Inter story drifts are estimates and its distribution along the height.
- Determination of force demands on brittle members, are axial force demands on columns, beam-column connections are moment demands
- Deformation demands of determination for ductile members.

- In location of weak points identification in the structure (or potential failure modes)
- Effort of an action strength deterioration of individual members on the behavior of structural system
- In plan or elevation identification of strength discontinuities that will lead to changes in dynamic characteristic's in the inelastic range.
- Verification of the completeness and adequacy of load path. Pushover analysis also exposes design weaknesses that may remain hidden in an elastic analysis. They are story mechanisms, excessive deformation demands, irregularities strength and overloads on potentially brittle members.

3.2 Material Specifications

1. **Steel Reinforcement**

   Modelled as *uniaxial bilinear stress-strain* model with kinematic strain hardening

Material Properties	
Modulus of Elasticity (kPa)	2.00E+08
Yield Strength (kPa)	250000
Strain Hardening Parameter (-)	0.005
Facture/Buckling Strain (-)	0.1
Specific Weight (kN/m3)	78

Table 3.1 Material Properties: Steel

Parameters required :

i. Modulus of elasticity - E_s

ii. Yield strength - f_y

iii. Fracture/buckling strain – ε_{ult}

iv. Strain hardening parameter – μ,

Where $\frac{Post\ Yield\ Stiffness}{Initial\ elastic\ stiffness}$

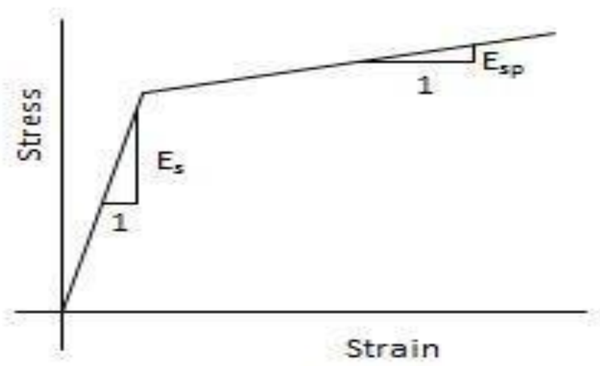

Fig. 3.2 Stress-Strain curve for Bilinear Steel

2. **Concrete**

Modeled as Non-linear material according to Mander et. al. [1988].

Material Properties	
Compressive strength(Kpa)	20000
Tensile strength (Kpa)	0
Strain at peak stress(m/m)	0.002
Confinement factor(-)	1.2
Specific Weight(KN/m3)	24

Table 3.2 Material Properties: Concrete

<u>Parameters required</u> :

i. Compressive strength - f_c

ii. Tensile strength - f_t

iii. Strain at peak stress – ε_c

iv. Confinement factor - k_c,

where $k_c \quad \dfrac{confined\ \ compressive\ \ stress}{unconfined\ \ compressive\ \ stress}.$

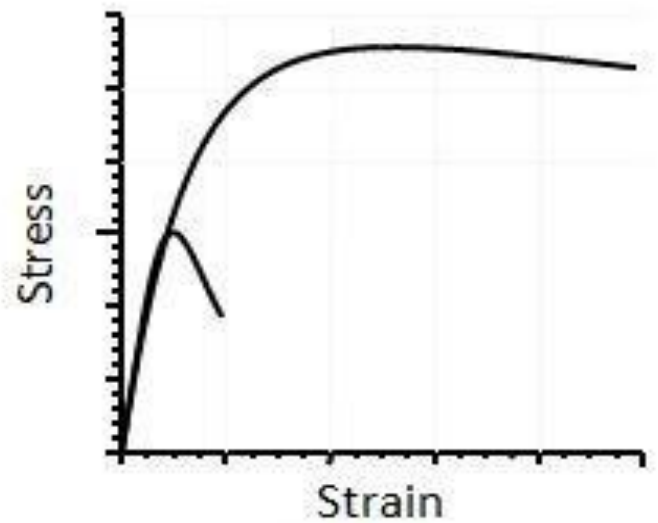

Fig 3.3 Stress-Strain curve for Mander's non-linear concrete

3.3 Data Compilation and Calculations:

Total sections of beams provided-

620mm x 400mm

430mm x 370mm

400mm x 300mm

400mm x 330mm

350mm x 200mm

Total sections of columns provided-

450mm x 275mm

337mm x 550mm

450mm x 550mm

525mm x 550mm

475mm x 575mm

Lumped mass is calculated and applied for each node which is the effective load acting on the node due to the dead weight of the floor slab and the infill walls.

Reinforcement in beam and column sections for the structure are calculated using STAAD.Pro using only gravity load condition with C 20 concrete and Fe250 steel reinforcement assumed in accordance with the expectation for a 15 year old building.

These sections are assigned to the simulation of the structure made in SAP-2000 and lumped masses are also assigned to each node. Thus the structure is simulated in SAP-2000 with 4 stories-8 bays-3 frames.

This structure is loaded from x-axis and y-axis to get separate performance curves for each axes. Incremental load (triangular loading) is applied to the structure

3.3.1 Calculation of Base Shear:

Clause 3.2.2.2 of EBCS-2013, the elastic response spectrum is defined by the expressions to find With the use of the software STAAD.Pro v8i the Base Shear was calculated in accordance with Euro code -8, and estimated to be **499.3kN**.

This base shear is shared amongst each floor as:

Loading along x-axis:

11.095 kN (Slab Level 1)

22.191 kN (Slab Level 2)

33.287 kN (Slab Level 3)

44.382 kN (Slab Level 4)

55.478 kN (Slab Level 5)

Loading along y axis:

3.6985 kN (Slab Level 1)

7.397 kN (Slab Level 2)

11.095 kN (Slab Level 3)

14.794 kN (Slab Level 4)

18.4926 kN (Slab Level 5)

3.3.2 Loading Phases:

x-axis loading-

Target Displacement: **0.600 m**

No. of steps: **1200**

y-axis loading-

Target Displacement: **0.600 m**

No. of steps: **200**

After loading the building along both the directions in the above discussed fashion the structure reached failure at little less than 525 mm during the x-axis loading and around 550 mm when loaded along y-axis as can be seen in the pushover plots.

3.3.3. Calculation of Seismic Weight:

Section	**Length**	**Number**	**Volume**
0.43x0.37	24.12	5	19.18746
0.40x0.33	24.12	5	3.18384
0.40x0.30	24.12	5	14.472
0.62x0.40	8.89	45	99.2124
0.35x0.20	2.95	10	2.065
0.57x0.47	15.0	4	16.074
0.28x0.45	15.0	9	17.01
0.55x0.52	15.0	4	17.16
0.55x0.34	15.0	2	5.61
0.55x0.45	15.0	8	29.7

(*Table 3.3 Beam and Column Section Details: Seismic Weight Calculation*)

Total volume = 223.6747m^3

Seismic weight due to dead load (beam + column) = (223.6747m^3) x (24kN/m^3) = 5368.2kN

Seismic weight due to dead load (slab) = (238.1m^2) x (3.7kN/m^2) = (880.97kN) x 4 = 5285.88kN

Seismic weight due to imposed load= (238.1m^2) x (4kN/m^2) x 0.5 x 3 = 1428.6kN

Hence, total seismic weight, W = 12082.62kN

3.3.4 Calculation of Target Displacement:

Calculation of K_e and V_y:

The nonlinear force-displacement relationship between base shear and displacement of the control node shall be replaced with an idealized relationship to calculate the effective lateral stiffness, K_e, and effective yield strength, V_y, of the building.

1. This relationship was bilinear, with initial slope K_e and post-yield slope α

2. Line segments on the idealized force-displacement curve was located using an iterative graphical procedure that approximately balances the area above and below the curve.

3. The effective lateral stiffness, K_e, was taken as the secant stiffness calculated at a base shear force equal to 60% of the effective yield strength of the structure.

4. The post-yield slope, α, was determined by a line segment that passes through the actual curve at the calculated target displacement.

5. The effective yield strength should not be taken as greater than the maximum base shear force at any point along the actual curve.

x-axis loading:

Now, from ASCE 41-06, the effective fundamental period,
where,

T_i; = elastic fundamental period (in seconds) in the direction under consideration calculated by elastic dynamic analysis;

K_i, = elastic lateral stiffness of the building in the direction under consideration calculated using the modeling requirements of Section 3.2.2.4;

K_e, = effective lateral stiffness of the building in the direction under consideration.

From x axis loading graph, we can see that slope k_e and k_i are almost equal. For our calculations we have taken the approximation to be negligible and hence, since k_e=k_i, we now have T_e=T_i.

i.e $T_e = T_i = 0.275$s

Now, using ASCE 41-06, target displacement can be calculated using,

$$\delta_t = C_0 C_1 C_2 S_a \frac{T^2}{4\pi^2} g$$

The coefficient C_0 relates the elastic response of an SDF system to the elastic displacement of the MDF building at the control node taken as the first mode participation factor. From Table 3-2 of ASCE 41-06, we can get C_0 as,

$C_0 = 1.35$.

Now, according to ASCE 41-06,

The value of "a" is equal to 130 for soil site class A and B, 90 for soil site class C, and 60 for soil site classes D, E, and F according to 3.3.3.3.2 of ASCE 41-06. Using expert opinion on the matter and referring suitable material on the subject the site class factor, a=60. The soil on site has been taken as belonging to "Class D" according to the parameters given in Clause 1.6.1.4.1 of ASCE 41-06.

And, according to Section 1.6.1.5.3 of ASCE 41-06, the generalized value of S_a can be found using either,

$$S_a = S_{xs}\left[\left(\frac{5}{B_1} - 2\right)\frac{T}{T_1} + 0.4\right] for\ 0 < T < T_0$$

$$or\ \ S_a = \frac{S_{xs}}{B_1}\ for\ T_o \leq T \leq T_s$$

$or \quad S_a = \frac{S_{xs}}{B_1 T}\ for\ T > T_s$, where $T_s = \frac{S_{x1}}{S_{xs}}\ and\ T_o = 0.2T_s$

And $B_1 = \frac{4}{[5.6 - \ln(100\beta)]}$

According to 1.6.1.5.3 of ASCE 41-06 due to absence of external cladding and presence of simple R.C frame , the damping of the structure is assumed to be 2%

Hence $\beta = 0.02$

$$B_1 = \frac{4}{5.6 - .693}$$

Since Walyta sodo is in zone- IV which fall under the category of high level of seismicity , according to the Table-1.6 of ASCE 41-06, $S_{xs} < 0.167$

Hence let us assume $S_{xs} = 0.165$

Since the effective fundamental time period is 0.275 sec we can assume $T_o \leq T \leq T_s$(Plateau region of the spectral curve)

Hence using $S_a = \frac{S_{xs}}{B_1} = \frac{0.165}{0.815} = 0.202454$

Also from graph , we get $V_y = 1300kN$

Total seismic weight of the building according to the calculations, $W = 12058.62\ kN$

According to Table 3-1 (ASCE-41-06)

$$C_m = 0.9$$

Hence, subsuming values in, R= $\frac{S_a}{V_y/_W} c_m$, we get

Or $R = \frac{0.202454}{1300/10320.68} X\ 0.9$

i.e, $R = 1.44655$

Substituting the values in the formula for C_1 we get,

$$C_1 = 1 + \frac{R-1}{aT_e{}^2} = 1 + \frac{1.44655 - 1}{60(0.275^2)} = 1.0984$$

Now $C_2 = 1 + \frac{1}{800}\left(\frac{R-1}{T_e}\right)^2 = 1.0033$

Using the above calculated values in the target displacement formula

$\delta_t = C_0 C_1 C_2 S_a \frac{T^2}{4\pi^2} g$

$$\delta_t = 1.35 * 1.0984 * 1.0033 * 0.202454 * \frac{0.275^2}{4\pi^2} * 9.81 = 0.0056m = 5.66mm$$

Hence the pushover curve for the structure with x-axis loading will be loaded for a displacement of 150% of $_t$ which is **8.48765mm** at the top node.

y-axis loading:

For y-axis loading, $C_1 = 1 + \frac{R-1}{a\,{T_e}^2},\ where\ a = site\ \ class\ factor\ and\ R = \frac{S_a}{V_y/W}\ C_m$

T_i; = elastic fundamental period (in seconds) in the direction under consideration calculated by elastic dynamic analysis;

K_i, = elastic lateral stiffness of the building in the direction under consideration calculated using the modeling requirements of Section 3.2.2.4;

K_e, = effective lateral stiffness of the building in the direction under consideration.

From x axis loading graph, we can see that slope k_e and k_i are almost equal. For our calculations we have taken the approximation to be negligible and hence, since k_e=k_i, we now have T_e=T_i.

i.e $T_e = T_i = 0.4238$s

Now, using ASCE 41-06, target displacement can be calculated using,

$$\delta_t = C_0 C_1 C_2 S_a \frac{T^2}{4\pi^2} g$$

The coefficient C_0 relates the elastic response of an SDF system to the elastic displacement of the MDF building at the control node taken as the first mode participation factor. From Table 3-2 of ASCE 41-06, we can get C_0 as,

$C_0 = 1.35$.

Now, according to ASCE 41-06,

The value of "a" is equal to 130 for soil site class A and B, 90 for soil site class C, and 60 for soil site classes D, E, and F according to 3.3.3.3.2 of ASCE 41-06. Using expert opinion on the matter and referring suitable material on the subject the site class factor, a=60. The soil on site has been taken as belonging to "Class D" according to the parameters given in Clause 1.6.1.4.1 of ASCE 41-06.

And, according to Section 1.6.1.5.3 of ASCE 41-06, the generalized value of S_a can be found using either,

$$S_a = S_{xs}\left[\left(\frac{5}{B_1} - 2\right)\frac{T}{T_1} + 0.4\right] for\ 0 < T < T_0$$

$$or\ \ S_a = \frac{S_{xs}}{B_1}\ for\ T_o \leq T \leq T_s$$

or $\quad S_a = \frac{S_{xs}}{B_1 T}\ for\ T > T_s$, where $T_s = \frac{S_{x1}}{S_{xs}}\ and\ T_o = 0.2T_s$

And $B_1 = \frac{4}{[5.6 - \ln(100\beta)]}$

According to 1.6.1.5.3 of ASCE 41-06 due to absence of external cladding and presence of simple R.C frame , the damping of the structure is assumed to be 2%

Hence $\beta = 0.02$

$$B_1 = \frac{4}{5.6 - .693}$$

Since Walyta sodo is in zone- IV which fall under the category of high level of seismicity, according to the Table-1.6 of ASCE 41-06, $S_{xs} < 0.16$

Hence let us assume $S_{xs} = 0.165$

Since the effective fundamental time period is 0.275 sec we can assume $T_o \leq T \leq T_s$(Plateau region of the spectral curve)

Hence using $S_a = \frac{S_{xs}}{B_1} = \frac{0.165}{0.815} = 0.202454$

Also from graph, we get $V_y = 1833kN$

Total seismic weight of the building according to the calculations, $W = 12058.62\ kN$

According to Table 3-1 (ASCE-41-06)

$$C_m = 0.9$$

Hence, substuting values in, R= $\frac{S_a}{V_y/_W} c_m$, we get

Or $R = \frac{0.202454}{1833/12058.62} X\ 0.9$

i.e, $R = 1..026$

Substituting the values in the formula for C_1 we get,

$$C_1 = 1 + \frac{R-1}{aT_e{}^2} = 1 + \frac{1.44655 - 1}{60(0.275^2)} = 1.0024$$

Now $C_2 = 1 + \frac{1}{800}\left(\frac{R-1}{T_e}\right)^2 = 1.000004705$

Using the above calculated values in the target displacement formula

$$\delta_t = C_0 C_1 C_2 S_a \frac{T^2}{4\pi^2} g$$

$$\delta_t = 1.35 * 1.0024 * 1.000004705 * 0.202454 * \frac{0.4328^2}{4\pi^2} * 9.81 = 0.0122274m$$

$$= 12.2274mm$$

Hence the pushover curve for the structure with x-axis loading will be loaded for a displacement of 150% of $_t$ which is 1**8.34mm** at the top node.

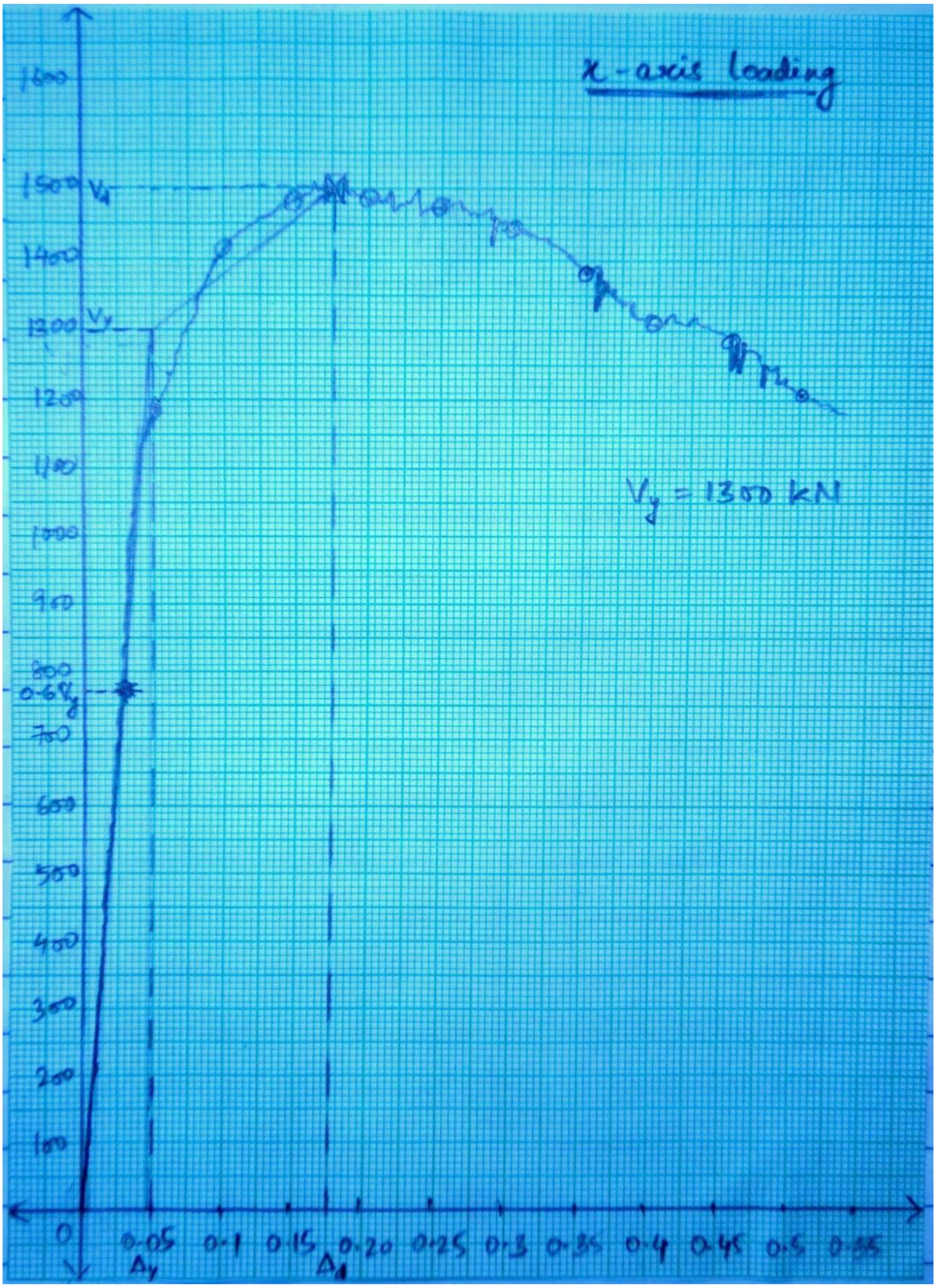

Fig. 3.4 Idealized pushover curve: x axis loading

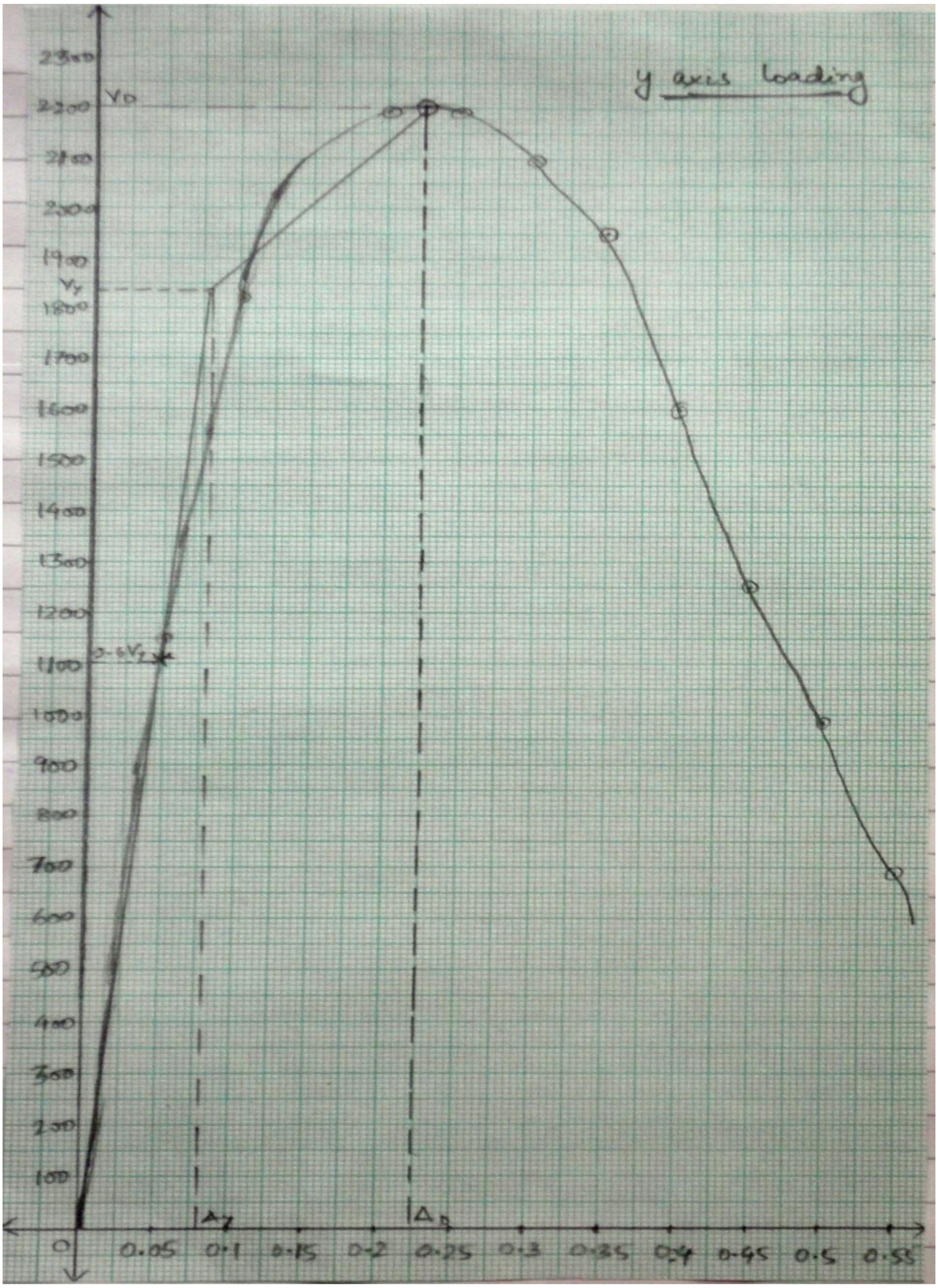

Fig 3.5 Idealized pushover curve: y axis loading

Chapter 4

RESULTS

4. Results and Discussions

4.1 RESULTS:

1. **<u>For x-axis loading</u>:**

3-D Rendering

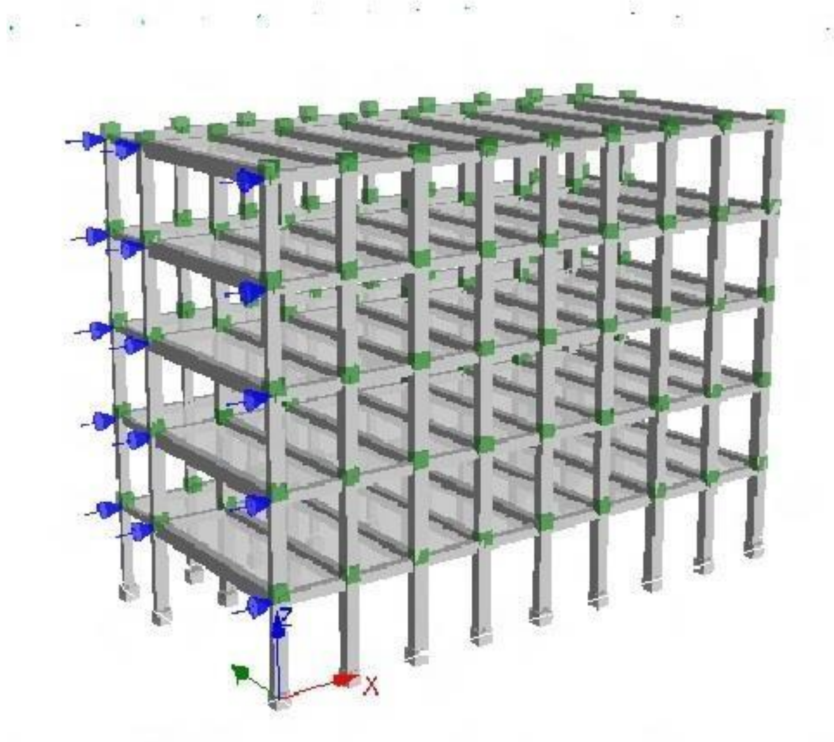

Fig 4.1 3-D rendering for x-axis loading

Roof Displacement versus Base Shear Plot

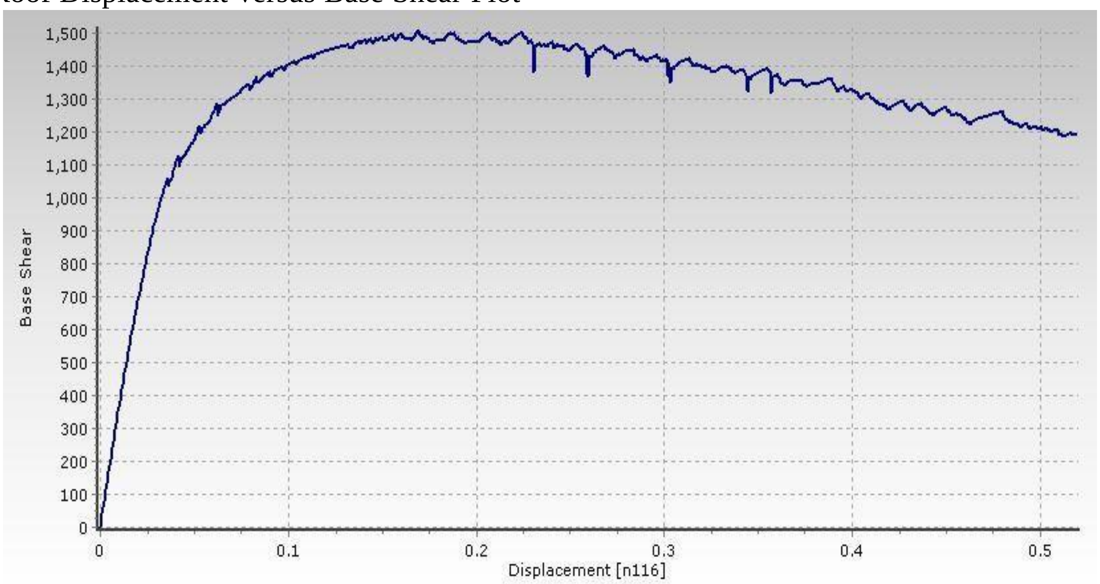

Fig 4.2 Capacity curve generated upon x-axis loading

Inter-story Drift versus Base Shear Plot

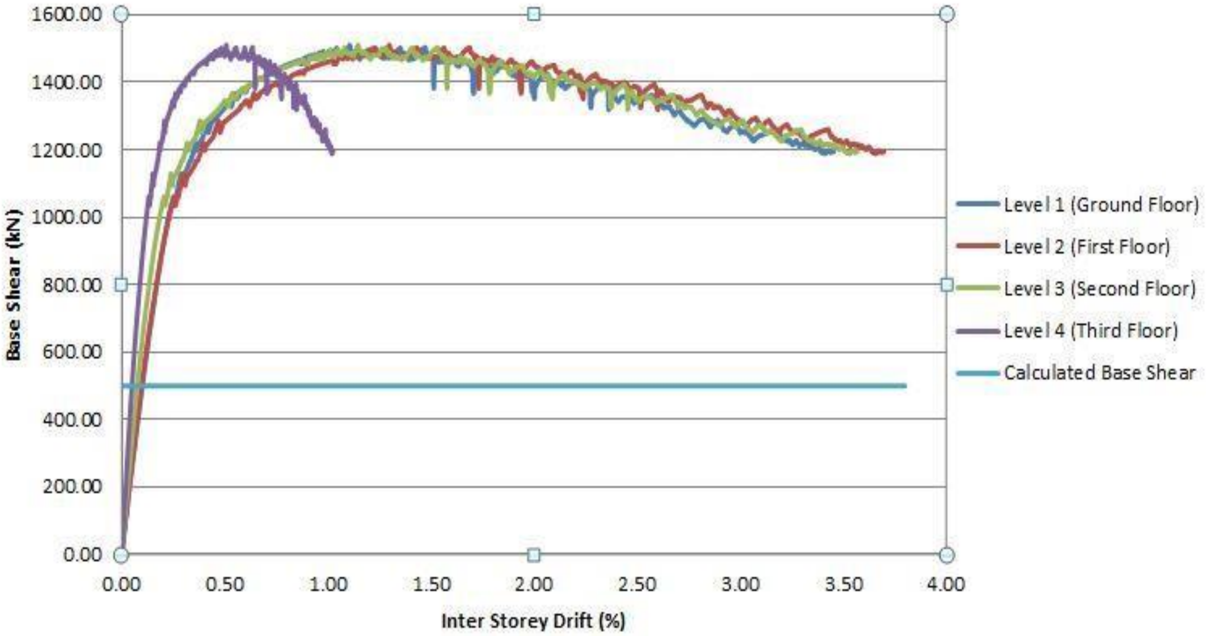

Fig 4.3 Inter-story Drift versus Base Shear Plot upon x-axis loading

2. **<u>For y-axis loading</u>:**
   3-D Rendering

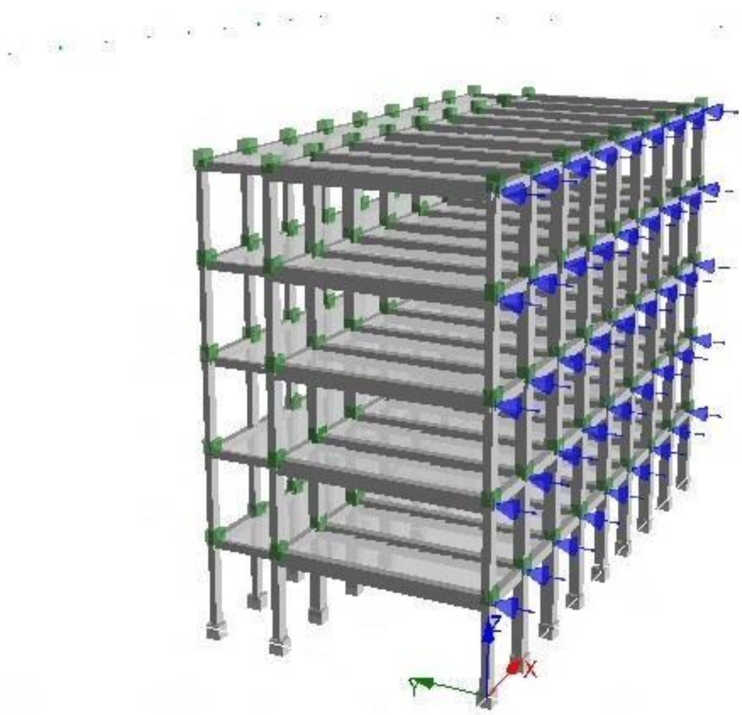

Fig 4.4 3-D rendering for y-axis loa

Roof Displacement versus Base Shear Plot

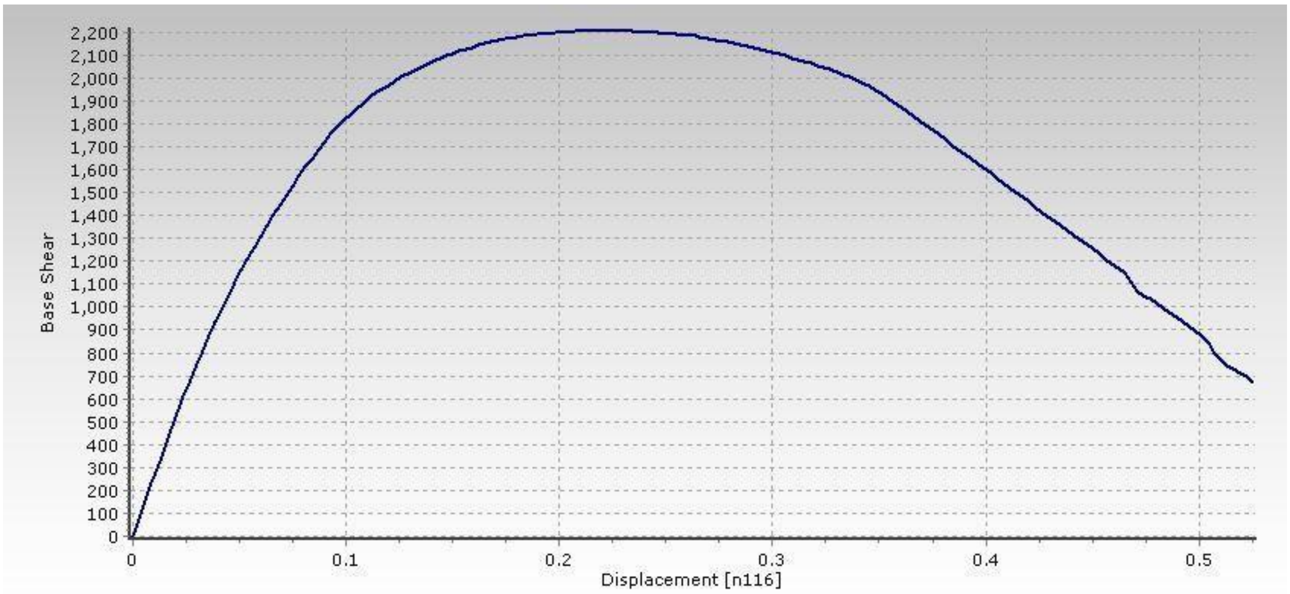

Fig 4.5 Capacity curve generated upon y-axis loading

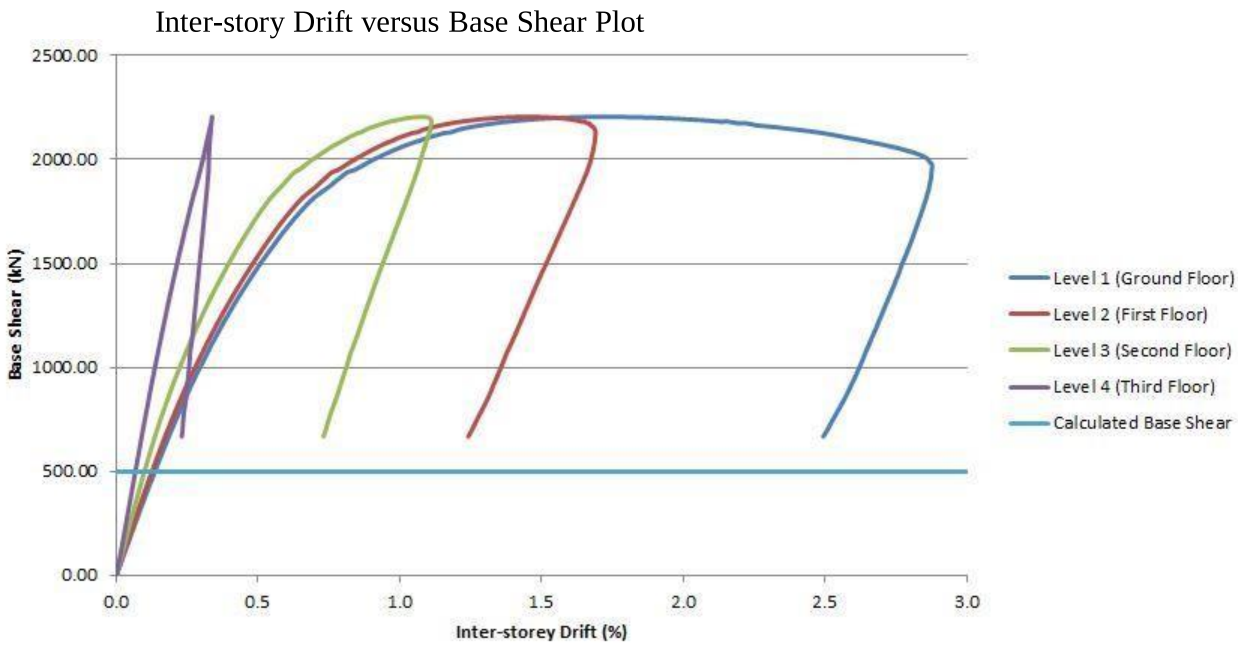

Fig 4.6 Inter-story Drift versus Base Shear Plot upon y-axis loading

The target displacement calculated in Chapter 3 in section 3.3.4 is used in SAP-2000 for both x-axis loading and y axis loading to generate pushover curves which indicate the behavior of the structure.

Pushover curves for calculated Target Displacements:

1. The maximum top node displacement given is **8.48765mm.**

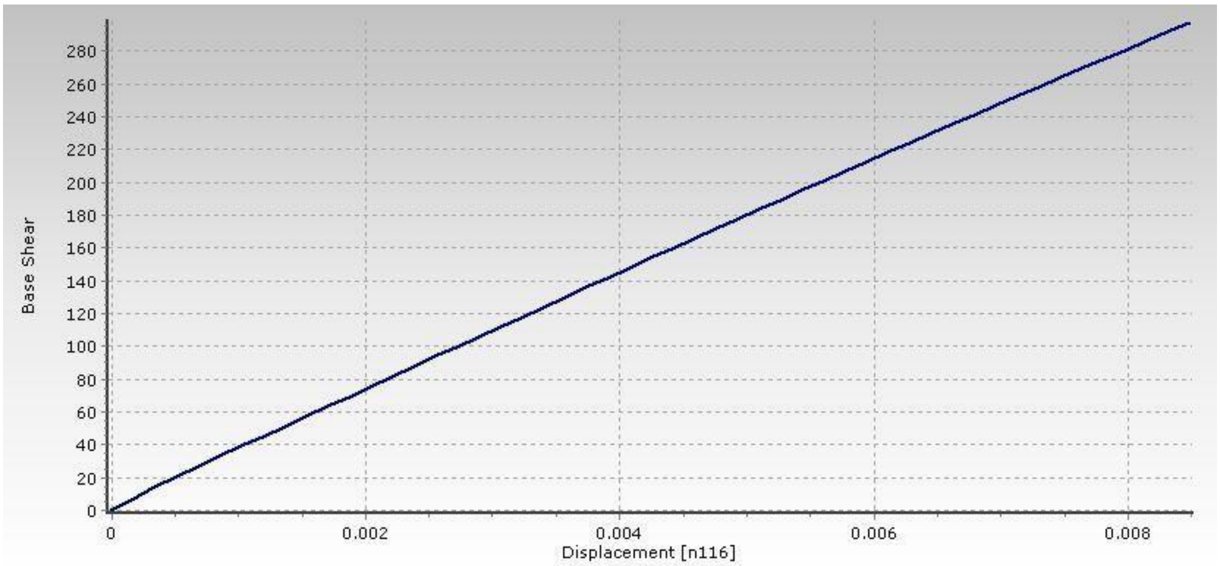

Fig 4.7 Pushover Curve for x axis loading up to target displacement

2. The maximum top node displacement given is **18.34mm.**

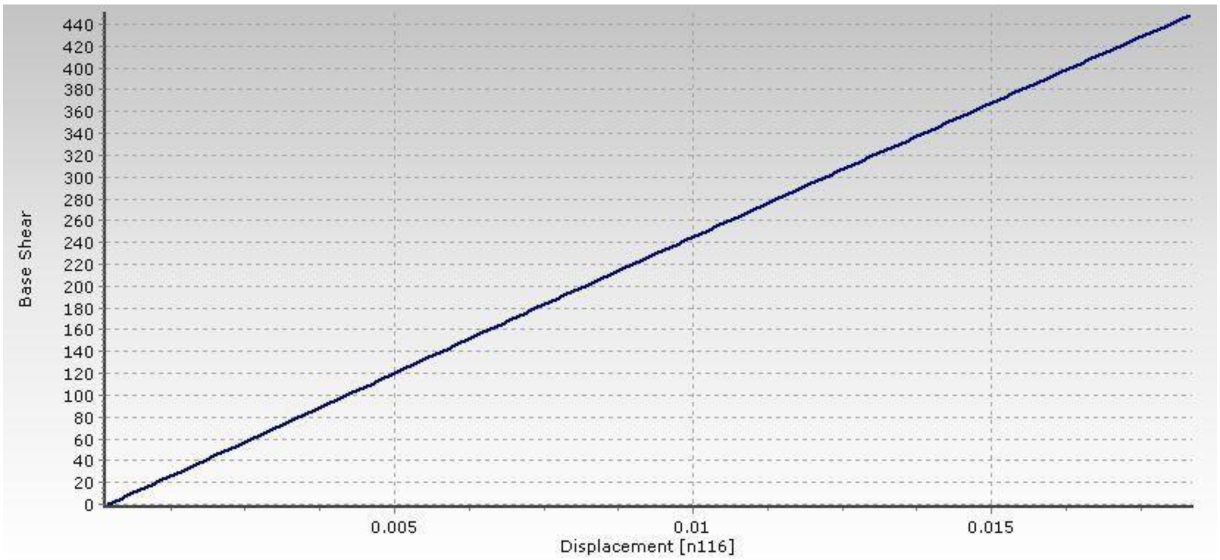

Fig 4.8 Pushover Curve for y axis loading up to target displacemet

4.2 DISCUSSION:

- The pushover analysis was an ideal method used to explore the non-linear behavior of the structure and for assessing the inelastic strength and deformation demands and for exposing design weakness.

- The materials assumed were (C20) Mender's concrete and (Fe250) bilinear steel.

- The performance criteria for the material in the simulation was: crushing strain limit for unconfined concrete- 0.0035; crushing strain limit for confined concrete-0.008; yield strain limit for steel- 0.0025; fracture strain limit for steel- 0.060.

- The pushover curve obtained upon loading the structure to collapse was converted to an idealized force-displacement plot.

- Target displacement is calculated according to displacement coefficient method.
- The structure analyzed to the target displacement limit has shown no failure.

- Hence according to this study, the building is completely safe and does not need to be retrofitted.

Chapter 5

CONCLUSION

5. Conclusion and Recommendations

5.1 CONCLUSION:

- The pushover analysis is a useful tool for assessing the inelastic strength and deformation demands and for exposing design weakness. The pushover analysis is a relatively simple way to explore the non-linear behavior of the structure.

- The pushover analysis is undertaken by loading the structure to the calculated base shear for limiting displacement, then the structure is pushed to a state of complete collapse and a pushover curve is obtained using SAP-2000

- Taking into account the low level of seismicity of Walito Sodo and the characteristic features of the structure and using ASCE 41-06, the target displacement is calculated.

- Upon loading the structure to the calculated base shear and limiting the displacement of control node, the pushover analysis reveals the structure is SAFE and hence the building does NOT need to be retrofitted.

5.2 FUTURE SCOPE OF STUDY:

An inclusion of shear failure limits in the performance criteria may lead to a better and more comprehensive understanding of the building's behavior.

Non-linear time history analysis can be used for the structure to have a more accurate assessment of the structure's capacity and understanding a more realistic demand scenario.

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